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Static and Dynamic Properties of Leighton Buzzard Sand from Laboratory Tests

Antonio Cavallaro
University of Catania, Italy

Michele Maugeri
University of Catania, Italy

Rosalia Mazzarella
University of Catania, Italy

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Antonio Cavallaro
Department of Civil and Environmental Engineering , University of Catania

Michele Maugeri
Department of Civil and Environmental Engineering , University of Catania

Rosalia Mazzarella
Department of Civil and Environmental Engineering , University of Catania

ABSTRACT

The Leighton Buzzard sand is an English sand commonly used at the University of Bristol to study soil-shallow foundation interaction (Maugeri et al., 1999a; 199b) and soil-retaining wall interaction (Carafa et al., 1998) by means of the shaking table. These experiments require an accurate definition of geotechnical characterisation of soil.
To define the mechanical behaviour of uncemented Leighton Buzzard sand a large number of static and dynamic laboratory tests were performed on dry reconstituted specimens. The static tests includes direct shear tests performed on specimens reconstituted by pluvial deposition method with different relative density D_r.
To evaluate the equivalent shear modulus G_{eq} and damping ratio D, resonant column tests and torsional shear tests were performed by mean of Resonant Column/Torsional Shear apparatus. The secant shear modulus G_s had evaluated during monotonic torsion tests by means of the same apparatus. Particular attention was devoted to the shear modulus at very small strain ($\gamma < 10^{-3}$ %) where the soil behaviour is supposed to be elastic and at intermediate strain level (from 10^{-3} % to 0.5 %) for simulating the prefailure deformation during the shaking table tests. The behaviour of soil at intermediate strain level is relevant for the serviceability limit state according the European Codes (EC7 and EC8). The dry reconstituted samples were subject to an initial anisotropic confining pressure able to simulate in situ effective stress status.
Finally, two expressions to allow the complete shear modulus degradation with strain level and the inverse variation of damping ratio with normalised shear modulus respectively were proposed.

INTRODUCTION

The Leighton Buzzard sand is an uncemented sand coming from a place near Bristol. This sand has been subject matter for precise investigations at the University of Bristol in relation to the study of soil shallow foundation interaction and soil retaining wall interaction by means of the shaking table (Taylor et al., 1994; Taylor and Crewe, 1996; Paolucci and Pecker, 1997; Carafa et al., 1998; Cassisi, 1998; Maugeri et al., 1999a; 199b).
A wide documentation by means of static tests was carried out from Stroud (1971).
The aim of this paper is to present an accurate static and dynamic geotechnical characterisation of this sand in order to allow the next analytical modelling of the mechanical behaviour.
To realise the geotechnical characterisation static and dynamic laboratory tests were performed. By means of direct shear test were computed the shear strength parameter φ, paying attention to the relationship with relative density D_r. On the other hand, the dynamic characterisation were determined using an apparatus which able to perform both torsional tests and resonant column tests (Lo Presti et al., 1993).

SOIL TESTED.

Tested sand is a natural uncemented sand namely Leighton Buzzard Sand.
The particle size distribution curve, obtained by means of the ASTM method for particle size analysis, was performed by a series of sieves and it is shown in Fig. 1.
The average particle size is D_{50} = 0.94 mm. The uniformity coefficient, defined as the ratio of D_{60} to D_{10}, is C = 2.128 and it points out the considerable homogeneity of the particle size.
The specific gravity of soil solid, determined with the ASTM standard test method (D 854), is G_s = 2.679.
The maximum dry density was determined by pluviation. The test was performed by using a sand spreader, available at the geothecnical laboratory of the Politecnico di Torino, developed by Miura and Toki (1982) and adapted by Lo Presti (1992). The maximum dry density value is γ_{max} = 17.94 kN/m^3.
On the contrary, minimum dry density was determined according to ASTM (4254-83) method and its value is γ_{min} =

15.06 kN/m^3.

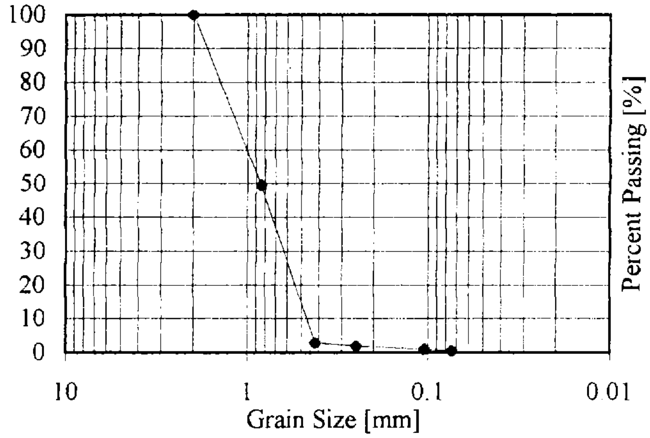

Fig. 1. Grading curve for tested sand.

DIRECT SHEAR TESTS

The tests were performed by means of direct shear test apparatus. They were used both standard Casagrande box (6×6×2 cm) and the larger one (10×10×2 cm).

Numerous direct shear tests, about sixty, were performed in such a way to determine the angles of shearing resistance in function of different relative density.

In each test, at least three samples were used to draw the shear envelope to check for test error or sample anomalies.

The specimens were reconstituted by means of the pluvial deposition procedure, developed by Lo Presti et al. (1992). This method allows obtaining fixed beforehand values of relative density, to reach a satisfying uniformity of the samples and to reduce the degree of trouble.

The sand is put into a container, supported by a trolley sliding along one direction. On the base of this container, the sand falls through a slit. The deposition is realised getting to swing the support at constant velocity while the slit is open.

It is possible to reconstitute samples with different relative density by placing the container at different height on the trolley, because relative density is correlated with the fall height of sand.

On the basis of the experimental results it was possible to obtain the following empirical correlation between D_r and h_d (height of deposition), Fig. 2:

$$D_r[\%] = 0.555 \cdot h_d[cm] + 14.7$$

before the application of the vertical load.

After preparing the sample, a constant vertical load was applied for a term of fifteen minutes. During this period, both upper and lower frames are strictly connected by means of four screws. The vertical stresses adopted (100 kPa, 200 kPa, 300 kPa) are maintained constant during the whole test time. After the rest period of fifteen minutes the screws are taken away and an electric motor produces the shear stress along the sliding surface. The advancement velocity adopted for tests on this sand is equal to 0.5 mm/min.

On the basis of the experimental results, it was possible to obtain the following empirical correlation between φ and D_r:

$$\varphi[°] = 0.238 \cdot D_r[\%] + 28.4$$

before the application of the vertical load, Fig. 3.

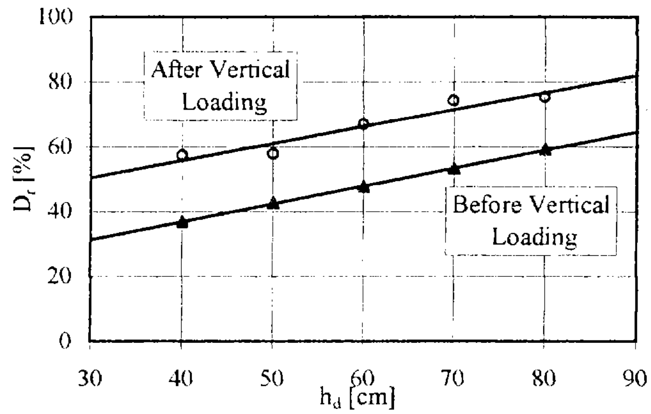

Fig. 2. Relative density vs. height of deposition.

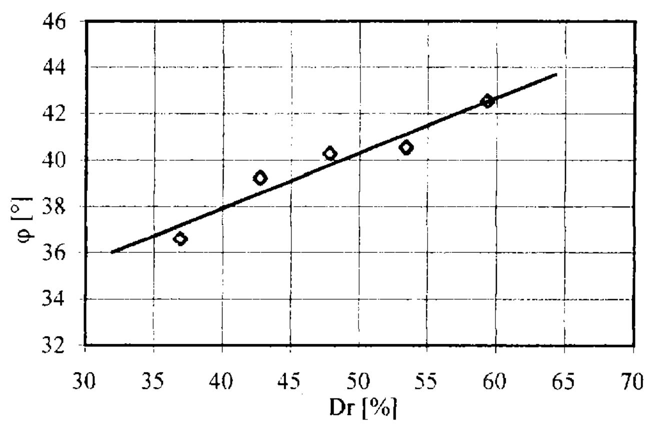

Fig. 3. Shear resistance angle vs. relative density.

RESONANT COLUMN\TORSIONAL SHEAR TESTS.

The equivalent shear modulus (G_{eq}) and damping ratio D were determined in the laboratory by means of a Resonant Column test (RCT) and cyclic loading torsional shear tests (CLTST) performed by means of a Resonant Column/Torsional shear apparatus (Lo Presti et al. 1993). Monotonic loading torsional shear tests (MLTST) were also performed using the same apparatus, obtaining the measurement of the secant shear modulus G_s.

G_{eq} is the unload-reload shear modulus evaluated from CLTST and RCT, while G_0 is the maximum value or also "plateau" value as observed in the G-log(γ) plot. Generally G is constant until a certain strain limit is exceeded. This limit is called elastic threshold shear strain (γ_t^e) and it is believed that soils behave elastically at strains smaller than γ_t^e. The elastic stiffness at $\gamma<\gamma_t^e$ is thus the already defined G_0.

The apparatus used is a fixed-free resonant column apparatus (Hall and Richart, 1963), adapted to perform monotonic and torsion shear tests (Pallara, 1995) on solid and hollow cylindrical specimens, Fig. 4. It enables the specimen consolidation under both isotropic and anisotropic stresses.
It is composed of a drive system, a support system, and a base plate, fixed on a concrete anchor block.
The solid or hollow cylinder specimen is fixed at the bottom and its constraint at the base is due to the friction existing between the specimen and the porous sintherized bronze stone (Drnevich et al., 1978).

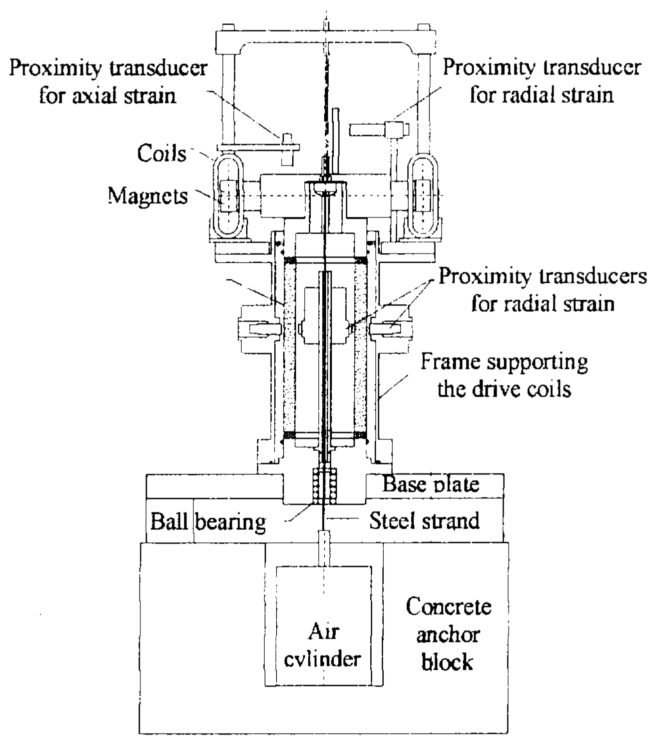

Fig. 4. Layout of the resonant column/torsional shear apparatus (after Lo Presti et al. 1993).

Torsional forces are applied at the top by means the drive system, realised in aluminium. It is an electrical motor constituted of four magnets connected with the top of the sample and eight coils placed on the inox steel annular base, which is strictly linked to the support system. The weight of the motor is counterbalanced by a spring. A programmable function generator (PGF) excites the electrical motor of Stokes.
The support system, in addition to permits the placement of the drive system, may possibly put the proximity transducers in and the filling in of water for saturated specimen tests.
It was realised a steel pressure cell, to permit the isotropic consolidation using an air pressure source controlled with a manual pressure regulator. The base and the top plates are connected by three vertical rod inside the cell.
In the present work hollow cylindrical specimens were reconstituted by using tapping (Drnevich et al., 1978), in order to obtain the required relative density and a good uniformity during the deposition.

The mold is assembled and a little depression is applied to let the membrane adhere to the inside surfaces.
The material is placed in the mold using a funnel-pouring device. The soil is placed as loosely as possible in the mold by leaving the soil from the spout in a steady stream, holding the pouring device upright and vertical, and maintaining constant the fall height. It is possible to obtain different values of relative density changing the height of deposition,.
In order to realise high values of relative density; it could be necessary to beat delicately the mold surface during the deposition. Each sample was reconstituted with fresh sand.
Each specimen was subjected to an isotropic load achieved in a steel pressure cell, using an air pressure source. An air cylinder (Bellofram type), fixed to the concrete anchor block and transmitted by a steel strand connected to the top, provided an additional vertical load. Such a system for anisotropic confining pressure was originally developed at the University of Texas at Austin (Isenhower et al., 1987) and permits the realisation of horizontal-to-vertical effective consolidation stress ratio K always less or equal to one.
Radial strain was measured using two couples of proximity transducers located at mid-height of the specimen and by monitoring both internal and external radius displacements.
The internal diameter variation can be measured at different heights by the couple of miniature proximity transducers, movable along a steel vertical rod. Membrane thickness is assumed constant.
The axial strain was measured by using a high-resolution proximity transducer, which monitors the aluminum top-cap displacement.
Shear strain was measured by monitoring the top rotation with a couple of high-resolution proximity transducers.
During a resonant column test, the proximity transducers are not able to appraise the value of the targets displacements, because of the high frequency of the oscillations. Then rotation on the top of the specimen is measured by means of an accelerometer.
For CLTSTs the damping ratio was carried out using the definition of hysteretic damping ratio (D) by:

$$D = \frac{\Delta W}{4\pi W} \tag{1}$$

in which ΔW is the area enclosed by the unloading-reloading loop and represents the total energy loss during the cycle and W is the elastic stored energy. For RCTs the damping ratio was determined using two different procedures: following the steady-state method, the damping ratio was obtained during the resonance condition of the sample; following the amplitude decay method it was obtained during the decrement of free vibration.
The laboratory test conditions and the obtained small strain shear modulus G_0 are listed in Table 1.
The dry reconstituted specimens were anisotropically submitted to a confining stress to simulate the real pressure conditions. Usually horizontal-to-vertical effective stress ratio K is equal to 0.5, except for two samples confined with a stress ratio K

equal to 0.8. Generally the same specimen was first subject to RCT, then CLTST to after a rest period of 30 minutes. In others cases the same specimen was first subject to MLTST, then RCT to after a rest period of 24 hrs. The size of hollow cylindrical specimens are Internal Diameter = 50 mm, External Diameter = 71 mm and Height = 142 mm.

Table 1. Test condition for Leighton Buzzard sand.

Test name	D_r [%]	e	σ'_{vc} [kPa]	K	G_o [MPa]	notes
B02RCT	65	0,591	49	0,5	104	
B02CLTST	68	0,584	49	0,5	91	after RCT
B03RCT	74	0,568	51	0,5	112	
B03CLTST	76	0,563	51	0,5	105	after RCT
B04RCT	53	0,646	150	0,5	135	
B04CLTST	56	0,620	151	0,5	147	after RCT
B05RCT	76	0,569	149	0,5	174	
B05CLTST	81	0,549	146	0,5	184	after RCT
B06MLTST	42	0.660	52	0.5	101	
B07MLTST	64	0.596	50	0.5	109	
B07RCT	71	0,594	51	0,5	174	after MLTST
B08MLTST	51	0.633	98	0.8	136	
B08RCT	53	0,626	98	0,8	151	after MLTST
B09MLTST	70	0.580	100	0.8	172	
B09RCT	70	0,579	100	0,8	173	after MLTST
B10MLTST	77	0.558	151	0.5	202	

The G_o values, reported in Table 1, indicate moderate influence of strain rate even at very small strain where the soil behaviour is supposed to be elastic. This different behaviour can be tentatively explained by considering that in this study the Leighton Buzzard sand showed a hardening behaviour as confirmed by the decrease of the void ratio from 0.646 in B04RCT to 0.620 in B04CLTST, fig. 5.

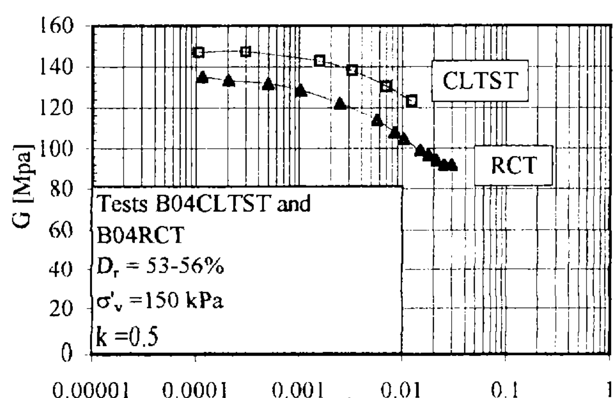

Fig. 5. G-γ curves from CLTST and RCT.

Figure 6 shows the results of RCTs normalised by dividing the shear modulus $G(\gamma)$ for the initial value G_o at very low strain.

The experimental results of specimens were used to determine the empirical parameters of the eq. proposed by Yokota et al. (1981) to describe the shear modulus decay with shear strain level:

$$\frac{G(\gamma)}{G_o} = \frac{1}{1 + \alpha\gamma(\%)^{\beta}} \qquad (2)$$

in which: $G(\gamma)$ = strain dependent shear modulus; γ = shear strain; α, β = soil constants.

The expression (2) allows the complete shear modulus degradation to be considered with strain level. The values of α = 20 and β = 0.9 were obtained for Leighton Buzzard sand.

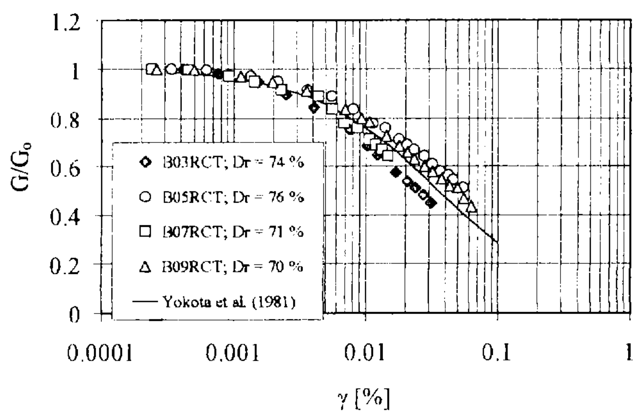

Fig. 6. G/G_o curves from RCT tests.

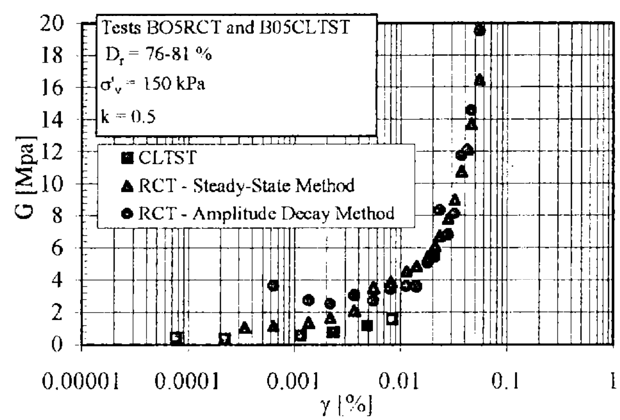

Fig. 7, Damping ratio from CLTST and RCT.

A comparison between the damping ratio values obtained from RCT and those obtained from CLTST is shown in Figure 7.

The damping ratio values obtained from RCT using two different procedures are similar even if for strain level less than 0.04 % higher values of D have been obtained from amplitude decay method. It is possible to see that the damping ratio from CLTST, at very small strains, is equal to about 0.43 %. Greater values of D are obtained from RCT for the whole investigated strain interval. Even if at very small strain the damping ratio assumes values so equal to zero.

As suggested by Yokota et al. (1981), the inverse variation of damping ratio with respect to the normalised shear modulus has an exponential form as that reported in Figure 8 for the Leighton Buzzard sand:

$$D(\gamma)(\%) = \eta \cdot \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \quad (3)$$

in which: $D(\gamma)$ = strain dependent damping ratio; γ = shear strain; η, λ = soil constants. The values of η = 134 and λ = 4.5 were obtained for Leighton Buzzard sand.

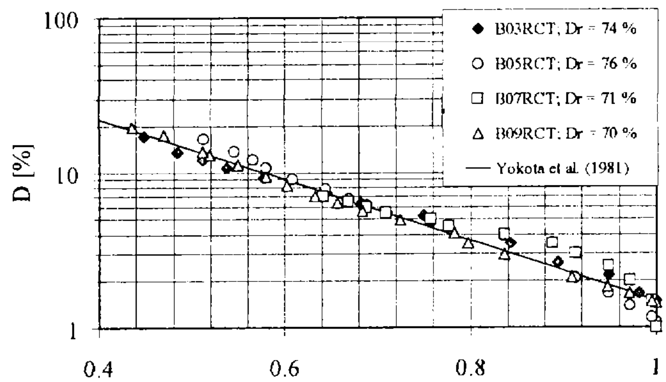

Fig. 8.D-G/G_o curves from RCT tests.

The equation (3) assume maximum value D_{max} = 22.15 % for $G(\gamma)/G_o$ = 0.4 and minimum value D_{min} = 1.48 % for $G(\gamma)/G_o$ = 1. Therefore, eq. (3) can be re-written in the following normalised form:

$$\frac{D(\gamma)}{D(\gamma)_{max}} = \exp\left[-\lambda \cdot \frac{G(\gamma)}{G_o}\right] \quad (4)$$

<u>Evaluation of G_0 from empirical correlations</u>

It was also possible to evaluate the small strain shear modulus by means of empirical correlation.

The empirical expression developed by Iwasaki et al. (1978), and here adopted, normalises G_o as regards the void ratio and the confining pressure:

$$G_o = 900 \cdot \frac{(2.17 - e)^2}{(1+e)} \cdot p_a \cdot \left(\frac{\sigma'_{ott}}{p_a}\right)^{0.43} \quad (5)$$

where: 900 = adimensional constant; e = void ratio; p_a = reference pressure (atmospheric pressure); $\sigma'_{oct} = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$ = octahedral pressure.

G_o, σ'_{ott} and p_a are expressed in the same unit.

The results of RCTs and CLTSTs obtained in the present work were compared with this relation, Table 2.

The results analysis shows that the equation (5) approaches in a reasonable way the experimental results. Higher scatters were observed for RCT results.

This behaviour can be explained by considering that the Iwasaki et al. (1978) equation was developed from cyclic loading torsional shear results.

Table 2. G_o from Iwasaki et al. (1978) equation.

Test name	Dr [%]	e	σ'_{vc} [kPa]	K	G_o (Iwasaki, 1978) [MPa]	G_o from tests [MPa]
B02RCT	65	0.591	49	0.5	88	104
B02CLTST	68	0.584	49	0.5	107	91
B03RCT	74	0.568	51	0.5	95	112
B03CLTST	76	0.563	51	0.5	114	105
B04RCT	53	0.646	150	0.5	129	135
B04CLTST	56	0.62	151	0.5	162	147
B05RCT	76	0.569	149	0.5	148	174
B05CLTST	81	0.549	146	0.5	181	184
B07RCT	71	0,594	51	0.5	88	174
B08RCT	53	0.626	98	0.8	122	151
B09RCT	70	0.579	100	0.7	134	173

Moreover, with regard to the resonant column tests, the higher differences can be observed in those samples that are been subjected to other tests (MTLST).

Moreover the normalised experimental results have been compared with the empirical correlation proposed by Jamiolkowski et al. (1991), that is valid for average pressure of consolidation σ'_m = 100 kPa:

$$G_o = 60 \cdot \frac{1}{e^{1.3}} \quad (6)$$

with G_o expressed in MPa. The equation (6) incorporate a term which expresses the void ratio.

G_o evaluated by laboratory tests has been normalised to the average pressure of consolidation of 100 kPa by means the expression:

$$G_{o,norm} = G_o \cdot \left(\frac{100}{\sigma'_m}\right)^{0.62} \quad (7)$$

where:

σ'_m is the average pressure of consolidation applied to the specimen.

In Table 3 the G_o values obtained by eq. (6) and the $G_{o,norm}$ values are showed.

The comparison between experimental values and empirical correlation has a good agreement. It is worthwhile to point out that the considered equation underestimate G_o.

Table 3. G_o from Jamiolkowski et al. (1994) equation.

Test name	Dr [%]	e	σ'_{vc} [kPa]	K	G_o Jamiolkowski, (1991) [MPa]	$G_{o\,norm}$ [MPa]
B02CLTST	68	0,584	49	0.5	121	140
B03CLTST	76	0,563	51	0.5	127	154
B04CLTST	56	0,620	151	0.5	112	113
B05CLTST	81	0,549	146	0.5	131	145

CONCLUSIONS

In this paper a geotechnical characterisation of Leighton Buzzard sand in static and dynamic field has been presented.
On the basis of pluvial deposition procedure and direct shear tests results empirical correlations between D_r and h_d and between φ and D_r are proposed.
Available data enabled one to define the small strain shear modulus trend and empirical equations are used to describe the G and D variation with strain level.
On the basis of the experimental results obtained, it is possible to draw the following conclusions:

- usually a good agreement on the small strain shear modulus G_o values are shown by RCT and CLTST.
- because a hardening phenomenon occurs during RCT, the G_o inferred from CLTST is about 15 % higher than RCT.
- at small strain damping ratio is pratically equal to zero.
- damping ratio values determined from RCT are greater than those obtained from CLTST.

differences between RCT and CLTST results are probably due to rate and/or frequency effects.
Finally the experimentally determined G_o, was compared to that inferred from available empirical correlations based on laboratory test results.

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